State of California

Business, Transportation and Housing Agency

Memorandum

MR. RAMIN RASHEDI
Division of Structure Design
Design Office 59-232

Attention Mr. Gary Blakesley

Date: December 29, 2000

File: 11-SD-5-KP 49.31
11-0301U1

Rte. 5/805 Separation (Widen)
Bridge No. 57-0512
Retaining Wall @ Abutment 5 Left (Widen)

From: **DEPARTMENT OF TRANSPORTATION**
ENGINEERING SERVICE CENTER
Division of Structural Foundations - MS 5
Office of Structure Foundations

Subject: Revised Foundation Recommendations

Introduction

The proposed Rte. 5/805 Separation (Br. No. 57-0512) is part of planned Route 5/805 Freeway improvements for the San Diego area. A Request for Final Foundation Recommendations (dated October 22, 1998) for the subject bridge was submitted to the Office of Structure Foundations (OSF) by Mr. Ramin Rashedi. Site specific ARS, liquefaction potential, and methods of liquefaction mitigation were requested in the above memorandum. A list of preliminary column/pile loads and shaft diameters were provided to OSF by Mr. Ramin Rashedi (dated December 18, 1998). As the 5/805 and 5/56 project has progressed, further revisions of the above pile load and shaft diameter list was sent to OSF including Revision 1 (dated February 24, 1999), Revision 2 (dated April 9, 1999), and Revision 3 (dated May 11 and 26, 1999). Bent pile diameters were confirmed by Mr. Ramin Rashedi (personal communication, September 1999). Abutment pile diameters and axial service loads were provided by Mr. Ramin Rashedi (February 1, 2000) who also requested P-Y curves or COM624 soil profile information at the abutments and final P-Y curves for the bents. Mr. Gary Blakesley (Bridge Engineer) provided final bottom of footing/pile cutoff elevations for the proposed bridge (Caltrans facsimile copy, dated March 24, 2000). P-Y curves were also requested by Mr. Earl Seaberg (Senior Bridge Engineer, Division of Structure Design) on February 24, 1999. In the same memorandum it was mentioned that in order to mitigate the effects of potential liquefaction, large diameter cast-in-drilled-hole piles (CIDH) would be used for structures at the 5/805 Interchange (Seaberg, February 24, 1999). In preliminary evaluations of the As Built Log of Test Borings performed by the Office of Geotechnical Earthquake Engineering (Jones and Abghari, February 10, 1999 and Perez-Cobo and Abghari, April 7, 1999), potentially liquefiable soils are estimated at approximately 7.6 to 9.1 m (25 to 30 ft) thick.

An additional Request for Revised Foundation Recommendations for the bents at the right side widening of the subject bridge was received November 13, 2000, from Mr. Gary Blakesley. Bent diameters for the right side widening were reduced from 2.4 m (8 ft) to a revised 2.1 m (7 ft). Pile loads, cutoff elevations, and finished grades would not change (Blakesley, November 13, 2000) from original conditions given previously. A revision is required to previous Foundation Recommendations for the Retaining Wall @ Abutment 5 Left (Widen) by Pratt (July 19, 2000). At this Abutment 5 Left Wingwall, the pile alternative option would require use of thicker HP305X110 (HP12X74) steel H-piles rather than the previously specified thinner HP250X85 (HP10X57) piles. This completely revised report is being sent for clarity and convenience regarding foundation recommendations. This revision supersedes the previous Foundation Recommendations (Pratt, May 5 and July 19, 2000) completed by the OSF.

Subsurface information was obtained by OSF drilling and sampling six - 94 mm diameter mud rotary borings which also involved extensive coring. Results from the field studies will be shown on the Log of Test Borings (LOTB). In addition to the recent field work, the As Built LOTBs for the Rte. 5/805 Separation (Br. No. 57-0512), Contract No. 11-022454, dated July 1964, contained additional site and subsurface information and will be included within the new contract plans.

Site Description

The existing abutments are dominantly founded in approach embankment fill material which ranges between approximately 15.7 m (51.5 ft) thick for the southern abutment (existing Abutment 1) and 14.0 m (46 ft) thick for the northern abutment (existing Abutment 5). Underlying native alluvium (Holocene and possible older Quaternary alluvium, undifferentiated) ranges from approximately 15.39 to 21.95 m (50.5 to 72 ft) thick for the proposed Right Side Widen and ranges from approximately 16.92 (estimated) to 20.27 m (55.5 to 66.5 ft) thick for the Left Side Widen. Alluvium thins to the south. The undulatory top surface of the underlying Eocene Ardath Shale was encountered from elevations ranging from –7.32 to an estimated –8.84 m (-24.0 to an estimated –29 ft) in the proposed Abutment 1 area widenings (south) to –11.73 to -12.68 m (-38.5 to -41.6 ft) at the proposed Abutments 5 and 6 area widenings (north).

Approach embankment fill material consists dominantly of firm to very stiff/loose to very dense, lean clay to clayey sand with minor scattered gravel and cobble-size soft mudstone rock fragments (up to 150 mm diameter) interlayered with silty sand and sandy silt with intermittent scattered gravel. Native material [mapped as Holocene alluvium and slope wash undifferentiated according to Kennedy (1975) and probably including some older alluvium at depth], can be divided into two units with the upper sediments consisting of dominantly loose to medium dense/soft to stiff, sandy lean clay, clayey sand, and rare fat clay interbedded with silty sand, sand, and sandy silt with intermittent scattered gravel. Calcite nodules, roots, and a thin [0.3 m (1 ft) thick peat lense (within Boring 99-5, Bent 2 Right Widen) were observed within these upper alluvial sediments. The underlying native alluvial unit [from 0.91 to 8.53 m (3 to 28 ft) thick] found below elevations ranging from approximately –3.96 to –9.14 m (-13 to -30 ft) consists of generally very dense/hard, gravel/cobble lenses with sand and clayey sand matrix interbedded and overlain by sand with 40% gravel and minor lean clay. Generally the extremely hard, subrounded to subangular gravel/cobbles [up to 155 mm (6 in) diameter] composed of metavolcanic, quartzite, and chert rock fragments directly overlie bedrock. Much of the loose and soft native material is considered potentially liquefiable and is being investigated by the Office of Geotechnical Earthquake Engineering (OGEE) for potential mitigation measures or adequacy of proposed mitigation measures. As mentioned earlier, final p-y (lateral resistance) curves are also being developed for use at proposed bridge support locations. The underlying Eocene Ardath Shale generally consists of interbedded very soft to moderately hard, mudstone, claystone, and siltstone. The formation is generally slightly weathered, slightly fractured, often thinly bedded, and contains occasional concentrations of pelecypod debris. The typical Ardath Shale in this area is partially underlain and interfingers with the Eocene Torrey Sandstone at Los Penasquitos Creek. The intertongueing sandstones representative of the Torrey Sandstone are composed of moderately hard to hard calcite cemented fine to medium sandstone and formational sand (uncemented, soil-like, very dense sand). The very soft to soft upper formational mudstones of the Ardath Shale [0.91 to 1.7 m (3 to 5.6 ft) thick], were considered to possess weak rock unconfined compressive strengths ranging from 120 to 180 psi. Below this upper zone, generally soft to moderately hard mudstone/claystone/siltstone/ and cemented sandstone (fairly strong rock) show unconfined compressive strengths from at least 250 to 300 psi and higher. The deepest boring for the bridge, Boring 99-2 (near proposed Bent 5 – Right Side Widen), extends to 51.76 m (169.8 ft) below the surface [elevation –40.45 m (-132.7 ft)]. Downhole P-S logging (compression and shear wave) showed that the better quality formational mudstones had shear wave velocities averaging 427 to 457 meters per second (1400 to 1500 fps) which appear to correlate with unconfined compressive strengths of at least 250 psi and higher.

Shear wave velocities ranging from 549 to 792 meters per second (1800 to 2600 fps) in pseudo-rock-like material correlated with unconfined compressive strengths of at least 300 psi and higher in the Ardath Shale below approximate elevation –24.38 m [(-80 ft), Br. No. 57-0512, Boring 99-2, Bent 5 - Right Side Widen] to -14.17m [(-46.5ft)., Br. No. 57-1069F, Boring 99-9, Bent 12, west of Abutment 1 – Left Side Widen for Br. No. 57-0512]. Beneath Bent 5 and Abutment 6– Right Side Widen (Boring 99-2), the upper competent mudstone tongue of the Ardath Shale thins to 7.92 m (26 ft) thick and interfingers with the Torrey formational sands (uncemented) and cemented sandstones. Similar thinning and interfingering of the mudstones take place near the southwestern portion of the subject bridge. The LOTBs should be reviewed for more specific details.

Surface Water and Scour

Surface water was often stagnant within Los Penasquitos Creek with only minor flow observed during the field investigation. Following a wet period on March 2, 2000, the author observed more substantial surface flow, however, flows were not enough to cause apparent substantial scour.

The proposed bridge will span Los Penasquitos Creek with existing supports, for Bent 3, proposed Bent 3 – Left Side Widen, and Bent 4 – Right Side Widen, located within the stream channel. It appears to OSF that remaining nearby supports are protected by embankment levees with rock slope protection. Only minor erosion or scour was found at the existing pier 2 for the nearby Los Penasquitos Creek Bridge, Br. No. 57-0779, during OSF's field investigation and brief survey of the stream bottom in the area. The As Built Boring B-1 (Br. No. 57-0779) shows up to 6.1 m (20 ft) of very loose to loose, silty sand with little clay binder below the channel surface on the east side of the bridge at existing Pier 2. The As Built LOTB for the Rte. 5/805 Separation (Br. No. 57-0512) reveals borings were drilled on the creek banks just out of Los Penasquitos Channel. However, Boring B-3 shows that sediments ranged from loose to medium dense/soft to firm and were composed of silty fine to medium sand with clay binder alternating with clayey sandy silt (consistency terms are modified to updated Caltrans standards). Most of OSF's recent laboratory gradations and plasticity indices reveal like sediments described on other As Builts for this area are often sandy lean clays and clayey sands which would result in reduced potential scour. OSF's survey of the bottom of the channel matched the elevation of the original Boring B-1 at approximately the same location. This may indicate that scour is not substantial at the site. According to the Preliminary Report (Wang, February 19, 1999) for the nearby Los Penasquitos Creek Bridge (Br. No. 57-0511L), total pier scour in this area is estimated at up to 2.5 m (8.2 ft).

Scour will not effect the foundations for the Rte. 5/805 Separation (Widen), with regards to axial support, which is gained within unscourable rock material. However, potential scour should be considered with regards to erosion and possible loss of lateral support within shallow soils at proposed in-channel bents. Due to the length of the piles, OSF feels that potential loss of significant lateral support is unlikely.

For further information, refer to Preliminary Investigations and Hydraulics reports in this area.

Ground Water

Boring 99-5 (Bent 2 – Right Side Widen) for the Rte. 5/805 Separation (Br. No. 57-0512) revealed static ground water at elevation +8.84 m (+29.0 ft) measured March 2, 2000 (shortly after rains). In the area of Boring 99-3 (nearby proposed Bent 7 on the south bank of Los Penasquitos Channel, N805/N5 Truck Connector), the water table may be estimated as high as +7.2 m (+23.6 ft) elevation based on P-wave velocity (measured June 30, 1999, during the dry season). The bottom of Los Penasquitos Creek in the area ranges from approximately +7.73 to +8.24 m (+25.4 to +27.0 ft) elevations and can be flooded. The ground water level fluctuated approximately 0.3 m (1 ft) during OSF recent investigation.

The As Built LOTB for the Rte.5/805 Separation shows ground water was encountered from approximate elevations +5.91 to +3.69 m (+19.4 to +12.1 ft) based on the City of San Diego datum, which requires a +2.45 m (+8.05 ft) add (Schuh, Caltrans Memorandum, March 7, 2000 and facsimile copy, February 14, 2000) to adjust to the current metric elevations (NAVD 88) upon which the recent plans and boring program are based. The adjusted to metric As Built elevations would then show ground water was encountered at elevations +8.36 to +6.14 m (+27.5 to +20.2 ft) for the earlier foundation investigation, with measurements taken during April 1962. The As Built LOTB for the Los Penasquitos Creek Bridge (Br. No. 57-0779) reveals ground water was encountered from elevations ranging from +6.58 to +6.40 m (21.6 to 21.0 ft). Again correcting English to metric (NAVD88) elevations would show ground water encountered from elevations +7.17 to +6.99 m (+23.5 to +22.9 ft) using an add of +0.588 m (1.93 ft).

Seismicity

See the memorandum (dated February 10, 1999) concerning Preliminary Seismic Design Recommendations sent to Mr. Earl Seaberg (Senior Bridge Engineer) from Mr. Ron Jones and Mr. Abbas Abghari. Final Seismic Design Recommendations and Lateral Resistance, p-y Curves will be submitted by the OGEE.

As mentioned above (Jones and Abghari, February 10, 1999) the proposed "structures are located approximately 5 km from the Newport-Inglewood-Rose Canyon fault which has a maximum credible earthquake moment magnitude of M=7.0 and based on the Caltrans California Seismic Hazard Map (Mualchin, 1995), these structures are within the peak horizontal bedrock acceleration zone of 0.5 g."

As mentioned above pseudo-rock-like material [Vs ranging from 549 to 792 meters per second (1800 to 2600 fps)] occurs below approximate elevations ranging from –24.38 m (-80 ft) in the area of Bent 5 – Right Side Widen and an approximately 5.2 m (17 ft) thick mudstone tongue below elevation –14.17 m (-46.5 ft) shows measured downhole shear-wave velocities in this range at nearby Boring 99-9 (proposed Bent 12, Br. No. 57-1069F) which was drilled west of proposed Abutment 1 – Left Side Widen.

Liquefaction

Liquefaction potential is considered moderate to high. Holocene and older Quaternary alluvium (undifferentiated) at the site is dominantly composed of loose to medium dense/soft to stiff, silty sand, sand, and sandy silt with intermittent scattered gravel interbedded with sandy lean clay, clayey sand, and rare fat clay. Ground water is also rather shallow [measured within the recent investigation at 6.43 m (21.1 ft) below the surface at Bent 2 – Right Side Widen (Boring 99-5) and estimated as shallow as 5.0 m (16.4 ft) below the surface at Bent 5 – Right Side Widen based on P-S logging]. Preliminary analysis (Jones and Abghari, February 10, 1999) estimates that the top 7.6 to 9.1 m (25 to 30 ft) of soils are considered potentially liquefiable. As mentioned above, final liquefaction potential is being determined by the OGEE.

Foundation Recommendations

The following recommendations are based on the Rte. 5/805 Separation (Widen), Br. No. 57-0512, General Plan (revised August 5, 1999), Foundation Plan (2 sheets, checked by S. Wang, October 13, 1998), the above mentioned memorandums and personal communications from Messrs Ramin Rashedi (Caltrans facsimile copy dated May 26, 1999, personal communications regarding pile loads and pile diameter, September 1999, and a memorandum supplying abutment pile diameters and service loads, February 1, 2000), and Gary Blakesley (Caltrans facsimile copy with final bottom of footing elevations, dated March 24, 2000). Revised bent pile diameters (with no change in finish grade or pile loads) for the right side widening were specified by Mr. Gary Blakesley (November 13, 2000). No revisions were required for the abutments and left side bridge widening. A revision in the thickness of steel H-piles is required for the pile alternative for the Retaining Wall @ Abutment 5 Left (Widen) due to corrosive soils and fill material according to Mr. Michael Tolin (August 17, 2000).

Fills can be placed in accordance with Section 19-6 of the Standard Specifications. End dumping is not permitted. At the Abutment 1 area, any settlement due to the addition of sliver fill widenings should be negligible in the foundation soils as existing embankment has been in place since 1964 and added load to the existing embankmentment will be minor. At the more significant Right Side Widen, additional fill is estimated at 7.92 m (26 ft) maximum height with a calculated maximum settlement of 350 mm (13.8 in). The settlement period is estimated at approximately 180 days, however the actual settlement period will be determined by the project engineer on the basis of settlement data in the field.

At the Abutment 6 (Right Side Widen), additional fill is estimated at 6.6 m (21.5 ft) maximum height. Existing fill for Abutment 5 has been in place since 1964 so settlement should be reduced from the original settlement of 0.95 ft measured at settlement platform #7 where 14.9 m (49 ft) fill height was added and the original settlement period was 180 days. Some additional fill has already been in place for months due to parallel ongoing improvements by San Diego County. Estimated settlement for the original conditions (encountered during OSF's recent field investigation) was calculated at approximately 228 to 343 mm (9 to 13.5 in). Our revised estimate due to recent changes in the area would be up to 228 mm (9 in) maximum possible settlement. Again, all fills can be placed in accordance with the Section 19-6 of the Standard Specifications. OSF recommends a fill settlement period of up to 90 days in this area; however, the actual settlement period will be determined by the project engineer on the basis of settlement data in the field.

Structure approach slab type N(9S) will be incorporated within the new bridge.

Plumb, 1.2m (4 ft) diameter, Cast-in-Drilled-Hole (CIDH) Piles can be used to support the widening at bridge abutments. Plumb, 2.1 m (7 ft) diameter drilled shafts will be used at the bents for the right and left side widening as shown below. Cast-in-Drilled-Hole Pile capacities were calculated using the Federal Highway Administration's Drilled Shaft Manual (Pub. No. FHWA-HI-88-042) published July 1988. Permanent casing is recommended to be placed into bedrock to facilitate construction of the drilled shafts, prevent caving of loose soils and gravel/cobble lenses into the pile borings, and seal off ground water from entering the pile borings. OSF feels that permanent steel casing can be emplaced at least near specified tip elevation using a vibratory hammer. In discussions between Mr. Ron Jones (Geotechnical Earthquake Engineering) and the author (for nearby Sorrento Viaduct, Br. No. 57-0513R/L, March and April, 2000) the practice of drilling ahead of the casing before dropping the casing into place is considered undesireable as caving of loose soils and gravel/cobble lenses would create voids between the casing and surrounding soil, thus compromising the lateral capacity of the pile. However, drilling slightly ahead of casing in the basal gravel/cobble lenses and within bedrock will probably be necessary. OSF assumes no additional axial geotechnical capacity for permanent steel casing that will be installed to aid in construction of CIDH piles shown below.

Rte. 5/805 Separation, Br. No. 57-0512 – right side widen:

Support Location/ Type & Diameter	Design Loading			Nominal Resistance		Intended Length of Rock Socket m (ft)	Bottom of Pile Footing/ Cutoff Elevation m (ft)	Permanent Casing Specified Tip Elevation m (ft)	Design Pile Tip Elevation m (ft)	Specified Pile Tip Elevation m (ft)
	Compression kN (tons)	Tension kN (tons)	Lateral kN (tons)	Compression kN (tons)	Tension kN (tons)					
Abut 1/ CIDH 1.2 m (4 ft)	**1275 (143.5)**			2550 (287)	0	3.66 (12.0)	+18.91 (+62.0)	-8.84 (-29.0)	-12.50(1) (-41.0)(1)	-12.50 (-41.0)
Bent 2/ CIDH 2.1 m (7 ft)				**9350 (1050)**		7.92 (26.0)	+9.16 (+30.1)	-8.84 (-29.0)	-16.76(1) (-55.0)(1)	-16.76 (-55.0)
Bent 3/ CIDH 2.1 m (7 ft)				**9350 (1050)**		7.62 (25.0)	+9.16 (+30.1)	-12.80 (-42.0)	-20.42(1) (-67.0)(1)	-20.42 (-67.0)
*Bent 4/ CIDH 2.1 m (7 ft)				**9350 (1050)**		6.40 (21.0)	+7.01* (+23.0)*	-13.41 (-44.0)	-19.81(1) (-65.0)(1)	-19.81 (-65.0)
Bent 5/ CIDH 2.1 m (7 ft)				**9350 (1050)**		6.55 (21.5)	+9.16 (+30.1)	-12.35 (-40.5)	-18.90 (1) (-62.0)(1)	-18.90 (-62.0)
Abut 6/ CIDH 1.2 m (4 ft)	**1525 (171.5)**			3050 (343)	0	3.35 (11.0)	+18.61 (+61.1)	-13.72 (-45.0)	-17.07(1) (-56.0)(1)	-17.07 (-56.0)

Notes: Design tip elevation is controlled by the following demands:(1) Compression;(2)Tension;(3)Lateral Loads

Rte. 5/805 Separation, Br. No. 57-0512 – left side (sliver) widen:

Support Location/ Type & Diameter	Design Loading			Nominal Resistance		Intended Length of Rock Socket m (ft)	Bottom of Pile Footing/ Cutoff Elevation m (ft)	Permanent Casing Specified Tip Elevation m (ft)	Design Pile Tip Elevation m (ft)	Specified Pile Tip Elevation m (ft)
	Compression kN (tons)	Tension kN (tons)	Lateral kN (tons)	Compression kN (tons)	Tension kN (tons)					
Abut 1/ CIDH 1.2 m (4 ft)	**1275 (143.5)**			2550 (287)	**0**	3.96 (13.0)	+18.45 (+60.5)	-9.45 (-31.0)	-13.41(1) (-44.0)(1)	-13.41 (-44.0)
Bent 2/ CIDH 2.1 m (7 ft)				**9350 (1050)**		7.62 (25.0)	+9.16 (+30.1)	-9.45 (-31.0)	-17.07(1) (-56.0)(1)	-17.07 (-56.0)
*Bent 3/ CIDH 2.1 m (7 ft)				**9350 (1050)**		7.62 (25.0)	+7.01* (+23.0)*	-12.80 (-42.0)	-20.42(1) (-67.0)(1)	-20.42 (-67.0)
Bent 4/ CIDH 2.1 m (7 ft)				**9350 (1050)**		6.71 (22.0)	+9.16 (+30.1)	-12.34 (-40.5)	-19.05(1) (-63.0)(1)	-19.05 (-63.0)
Abut 5/ CIDH 1.2 m (4 ft)	**1525 (171.5)**			3050 (343)	**0**	3.96 (13.0)	+18.15 (+59.5)	-13.44 (-44.0)	-17.40(1) (-57.0)(1)	-17.40 (-57.0)

Notes: Design tip elevation is controlled by the following demands:(1) Compression;(2)Tension;(3)Lateral Loads

*Ground elevation at the bottom of Los Penasquitos Channel was surveyed in the area (based on NAVD 88 datum) at +7.44 m (+25.4 ft) on east side of the NBND Rte. 805 bridge (Los Penasquitos Creek Bridge, Br. No. 57-0779, just east of pier 2). Pile cutoff elevations for supports within the Channel for the proposed widening of Br. No. 57-0512 are approximately estimated at 0.61 m (2 ft) below this surveyed elevation. Proposed supports will be just downstream of the above surveyed elevation.

When pile nominal resistance in tension is provided by DSD, OSF can then provide design pile tip elevations in tension. Also, if the pile tip elevation is controlled by lateral demands, the designer is responsible to present correct foundation data, governed by lateral control, on the foundation plans.

Axial compression values noted in the tables above are based on skin friction only within the rock. End bearing was not considered due to working below the water table and the possibility that cleaning out the bottom of pile borings effectively may be rather difficult at depth and may make it difficult to realize substantial end bearing using Caltrans standard pile vertical deflection criteria of 13 mm (0.5 in).

Bedrock topography (top of rock) is often quite variable across short lateral distances. Due to this fact, the pile data table above includes intended length of the rock socket at each support. The intended length of the rock socket should be measured from the bottom of the permanent casing down to the pile specified tip elevation. OSF feels that permanent casing should be seated into rock approximately 0.61 m (2 ft). If the bedrock slope is steeper than expected, the permanent casing may need to be seated slightly deeper to seal out water and potential caving soils.

Retaining Wall @ Abutment 5 Left:

A separate Request for Foundation Recommendations (Blakesley, personal commun., May 25, 2000) for the retaining wall (Abutment 5 Left wingwall) was submitted to the Office of Structure Foundations (OSF). The retaining wall will be located on the northwest end of the Rte. 5/805 Separatior –Widen (Br. No. 57-0512) and will retain proposed additional northbound Rte. I-5 embankment materia above the existing northbound Rte. 805 Freeway according to the Foundation Plan No. 2 (Br. No. 57-0512, revised May 12, 2000) which displays the wingwall layout. As mentioned earlier, a revision is required to the previous Foundation Recommendations for the Retaining Wall at Abutment 5 Left by Pr (July 19, 2000). At this Abutment 5 Left Wingwall, the pile alternative option would require use of thicker HP305X110 (HP12X74) steel H-piles rather than the previously specified thinner HP250X85 (HP10X57) piles. According to Mr. Michael Tolin of the Corrosion Technology Branch (August 17, 2000), thicker steel H-piles are required due to corrosive site conditions. In order to maintain a 75-year design life for the retaining wall, the author and Mr. Tolin agreed that additional sacrificial steel provide by HP305X110 (HP12X74) piles is considered satisfactory for use at the site.

No new additional foundation investigation was required at the site of the proposed wall as the recently completed foundation investigation for the bridge (completed January 5, 2000) is considered adequate by OSF.

Generally, sediments beneath the proposed wall footprint at the site consist of embankment fill material which will range between approximately 10.45 to 8.69 m (34.3 to 28.5 ft) thick. The above embankment material is underlain by native alluvium which is underlain by Eocene Ardath Shale, and interfingering Eocene Torrey Sandstone as described previously.

Liquefaction

As mentioned above, liquefaction potential is considered moderate to high. Holocene and older Quaternary alluvium (undifferentiated) at the site is dominantly composed of loose to medium dense/soft to stiff, silty sand, sand, and sandy silt with intermittent scattered gravel interbedded with sandy lean clay, clayey sand, and rare fat clay. Ground water is also estimated as shallow as 5.0 m (16.4 ft) below the surface at nearby Bent 5 – Right Side Widen based on P-S logging]. Preliminary analysis (Jones and Abghari, February 10, 1999) estimates that the top 7.6 to 9.1 m (25 to 30 ft) of soils are considered potentially liquefiable. As mentioned above, final liquefaction potential is being determined by the OGEE and may control foundation type selection for this retaining wall (a pile supported wall may be required).

The following recommendations are based on Foundation Plan No. 2 for the adjacent Rte. 5/805 Separation (Widen), Br. No. 57-0512, (sheet 2 of 2, revised May 5, 2000), the Abutment 1 and 5 Left Details No. 2 (Br. No. 57-0512, received 7-5-2000), and sporadic discussions with Messrs Ramin Rashedi and Gary Blakesley (Structure Design Engineers) from May to July, 2000.

The Retaining Wall (a Type I retaining wall) is approximately 16.180 m (53.1 ft) in length and var from 3.6 to 1.8 m (11.8 to 5.9 ft) in height.

The Retaining Wall at Abutment 5 Left for the Rte. 5/805 Separation (Widen), Br. No. 57-0512, might be supported by Standard Type 1 wall spread footings placed within existing embankment materi at elevations shown on the Foundation Plan No. 2 (revised May 5, 2000). Following footing excavation, exposed material should be compacted to 95% R.C. (relative compaction) at footing grade. The maxim allowable bearing for the retaining wall should not exceed 145 kPa (1.5 TSF).

As an alternative foundation for the wing wall (if OGEE determines that liquefaction potential controls foundation type selection), steel H-piles [HP305X110 (HP12X74)], 400 kN (45 ton) design load, can be recommended for wall support as indicated below. Heavier steel sections are recommended here due to anticipated hard driving conditions through cobble/gravel zones and very dense sand. Corrosive soils tested at the site require that additional sacrificial steel be provided to protect the structural integrity of the piles. The above heavier steel section, which is commonly used for 625 kN (70 ton) design load piles, contains the additional sacrificial steel required at the site. Predrilling of the embankment is required down to elevation +8.53 m (+28 ft) before pile installation.

Retaining Wall at Abutment 5 Left (Widen), Br. No. 57-0512:

Bottom of Pile Footing Elevation m (ft)	Approximate Begin Pile Bearing Elevation m (ft)	Design Pile Tip Elevation m (ft)	Specified Pile Tip Elevation m (ft)
+17.493 (+57.4)	+1.52 (+5.0)	-8.84(1) (-29.0)(1)	-8.84 (-29.0)
+18.093 (+59.4)	+1.52 (+5.0)	-8.84(1) (-29.0)(1)	-8.84 (-29.0)
+18.693 (+61.3)	+1.52 (+5.0)	-8.84(1) (-29.0)(1)	-8.84 (-29.0)
+19.293 (+63.3)	+1.52 (+5.0)	-8.84(1) (-29.0)(1)	-8.84 (-29.0)

Notes: Design tip elevation is controlled by the following demands:(1)Compression;(2)Lateral Loads

If pile tip elevation is controlled by lateral demands, the designer is responsible to present correct foundation data, governed by lateral control, on the foundation plans. OSF feels that alluvial soils may be potentially liquefiable above Approximate Begin Pile Bearing Elevation which is estimated at +1.52 m (+5.0 ft). All elevations are based on the current metric NAVD 88 datum.

Constructability

As mentioned above, OSF recommends installation of permanent casing to be placed into bedrock to prevent caving of loose soils and gravel/cobbles lenses into pile borings and help seal off ground water from entering the excavations once seated into rock. OSF and the OGEE feel that using a vibratory hammer to place steel casing down to a level close to casing specified tip elevation would facilitate pile construction and effectively reduce creation of voids along the pile length by undesireable caving of loose soils and subbrounded cobble/gravel material. Drilling ahead of the casing, especially within the upper loose/soft soil zones should be avoided to reduce caving and creation of voids, thus compromising lateral pile capacity. OSF anticipates center relief drilling to facilitate casing advancement. Hard slow drilling [through hard metavolcanic cobble zones (cobbles up to 155 mm diameter), cobble-size mudstone rock fragments, and bedrock] is anticipated during installation of permanent casing and CIDH piles (rock sockets). Drilling ahead of casing may be required, in order to advance casing within the lower gravel/cobble lenses and within bedrock. Once casing is seated into bedrock, drilling for the rock sockets can be completed

The Caliper log within Boring 99-2 (proposed Bent 5 -- Right Side Widen) which was an uncased hole, shows that caving happens readily within shallow loose/soft often saturated alluvium and within the sand and gravel/cobble lenses overlying bedrock.

Ground water should be anticipated at relatively shallow depths. Static ground water was measured at elevation +8.84 m (+29.0 ft) within Boring 99-5 (Bent 2 – Right Side Widen) and within nearby Boring 99-3 (proposed Bent 7 for the N805/N5 Truck Connector), the water table could be estimated as high as +7.2 m (+23.6 ft) elevation based on P-wave velocity, measured during the dry season. The wet method is advised for CIDH pile construction. The bottom of all excavations should be cleaned of loose debris before placing concrete.

Clay mineralogy within formational material appears sensitive to the introduction of fresh water, which could cause swelling of clays and slicking of borehole walls, resulting in reduced pile/soil skin friction capacity. OSF feels that a mud/polymer expert should be consulted and be available to the contractor to advise on proper drilling fluid/slurry chemistry in order to prevent clay swelling. OSF feels that seating permanent casing into the formational mudstones/claystones/siltstones should help seal off ground water from reacting with the formational clays.

Regarding the Abutment 5 Left wingwall, possible spread footing foundations would be well above static ground water level. Settlement magnitude from the additional sliver fill should be minor. OSF suggests a settlement waiting period of up to 90 days in this area. The actual waiting period shall be determined by the Project Engineer on the basis of settlement data in the field. The purpose of the proposed retaining wall is to keep added approach embankment for the Rte. 5/805 Separation (Left SideWiden) from encroaching on the adjacent northbound Route 805. At the completion of the settlement period, when settlement has essentially ceased to tolerable levels, the wall and bridge abutment widening can then be constructed.

If the pile alternative is used for the Abutment 5 Left wingwall, as mentioned above, predrilling through fill material is required and will help simplify pile installation. Hard driving is anticipated near specified tip elevation within dense gravel/cobbles zones and possible very dense sand (formational sand) or soft mudstone/claystone/siltstone. Also, some dense sand lenses may be encountered at more shallow depths within alluvium. Ground water should be anticipated at relatively shallow depths. Pile tips will be well below static ground water level.

Corrosiveness

Laboratory tests of one composite soil sample taken from Boring 99-3 [depths 3.05 to 5.03 m (10 to 16.5 ft)] show fill has a high sulfate content (5030 ppm) and much lower chloride content (190 ppm). This high sulfate content is considered corrosive. Laboratory tests of composite soil samples (taken within Boring 99-1 for nearby Retaining Wall No. 524) immediately south of the bridge, indicate that fill and native material are corrosive. Corrosion tests on embankment fill show a pH of 7.48, minimum resistivity of 475 ohm-cm, sulfate and chloride content were measured at 5730 and 760 ppm, respectively. Corrosion tests on alluvial material show pH ranges from 7.48 to 7.98, minimum resistivity ranges from 475 to 746 ohm-cm, sulfate and chloride content were measured at 6000 to 360 ppm and 230 to 150 ppm, respectively.

Caltrans Corrosion Technology Branch has provided detailed corrosion review and corrosion recommendations for both the Rte. 5/805 Separation – Widen, Br. No. 57-0512 (Tolin, June 27, 2000) and the Retaining Wall at Abutment 5 Left – Widen (Tolin, August 17, 2000). The above memorandums should be consulted for corrosion recommendations regarding CIDH piles (cased and uncased), pile caps, walls, footings, and sacrificial steel thickness for alternative H-pile support at the Retaining Wall at Abutment 5 Left – Widen.

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If you have any questions, please call Joe Pratt at (213) 620-2001 or Richard Fox at (916) 227-7085.

Report by:
Joseph S. Pratt

JOSEPH S. PRATT, C.E.G. No.2141
Associate Engineering Geologist

REGISTERED GEOLOGIST
JOSEPH S. PRATT
No. 2141
Exp. 05/31/01
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

c: R.E. Pending File
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